MATILIJA DAM
Ecosystem Restoration Feasibility Study

Appendix B
STRUCTURAL EVALUATION

September 2004

Executive Summary

Matilija Dam is a concrete, thin arch dam on Matilija Creek, a tributary to the Ventura River. Since its construction in 1946-1947, the dam has been exposed to adverse internal and external conditions that have affected its operation and safety. The purpose of this structural evaluation is to determine the existing conditions of the dam as related to the overall Matilija River Environmental Restoration Project. This evaluation includes a structural description of project features, a review of previous studies and modifications, a projection of future uses and condition, and a comparison of structural conditions with current design criteria. Additionally, a hazard classification was performed to qualitatively evaluate the risks associated with potential dam failure.

Concrete in the dam has experienced excessive deterioration due to *alkali-silica reaction (ASR)*. Concrete sampling and testing performed in past studies have shown a decrease in the concrete material properties for the upper portions of the dam. The deterioration is expected to spread to the lower portions of the dam as pressures confining the ASR are relieved through chemical expansion. Thus, the material properties of the concrete in the dam will continue to degrade for the remaining life of the structure.

The loads acting on Matilija Dam have increased dramatically since its original design. Sediment has filled in approximately 93% of the reservoir and acts against the upstream face of the dam. It is estimated that the sediment will reach spillway crest elevation by the year 2020. The original design included a probable maximum flood (PMF) peak inflow of 60,000 cfs. The most current structural analysis used a PMF peak inflow of 76,108 cfs, which results in 16.0 feet of head over the modified spillway crest. The earthquake induced ground accelerations have increased from the original ground acceleration of 0.1g to an updated Peak Horizontal Ground Acceleration (PHGA) of 0.7g for the Maximum Credible Earthquake (MCE). Chemical expansion due to ASR has increased the internal stresses within the structure (cracks develop if the stresses generate exceed the tensile strain capacity of the concrete). The developed stresses with the combination of other static and dynamic loads will impair the structural integrity of the dam. As a result of these load increases, the arch dam has been modified twice (lowering and widening the spillway) in order to maintain adequate factors of safety.

Matilija Dam is no longer operated for its original purpose. Originally, Matilija Dam provided groundwater recharge for the Ojai valley and flood control for the communities downstream of the project. Due to the spillway modifications and sediment deposition, the dam no longer has discharge capacity for groundwater recharge, or adequate storage for flood control. All flows are currently discharged into the river downstream of the dam. There are no current plans to expand the current operation.

Matilija Dam is a *high hazard dam*, with potentially substantial consequences in the remote event of a dam failure. Matilija Dam is categorized as high hazard based on the almost certain loss of life, the disruption of critical facilities and access, major damages

to public and private property, and extensive mitigation required for environmental damages. It should be noted that if the “No Action Alternative” is chosen as a result of this feasibility study, a risk assessment study might be advisable to determine actual risks and consequences. The classification of Matilija Dam as a high hazard dam in no way implies that there are structural deficiencies that render the project unsafe, and all previous studies conclude otherwise.

Analyses conducted by consultants to the County of Ventura have shown that Matilija Dam is adequately stable. The reports recommend continued operation with periodic inspection, and future concrete sampling and testing. Applying U.S. Army Corps of Engineers criteria to results of previous studies, Matilija Dam generally meets current safety criteria for arch dams. If the “No-Action” Alternative is chosen for this feasibility study, it is recommended that a finite element method (FEM) analysis be completed for the Operating Basis Earthquake (OBE) in conjunction with a comprehensive risk assessment study.

Based on the existing information provided by the County of Ventura and other sources, it is believed that Matilija Dam could remain in service for an additional 50 years. However, since the quality of concrete will decrease, and the loading due to silt will increase, modifications may be required in order to maintain an adequate level of safety for the dam. The scope of such modifications is dependent upon the actual rate of concrete deterioration and silt sediment deposition. The County of Ventura has stated that there are no plans for modifying the project from its current condition. Thus, the baseline condition is to assume that the dam will remain in its current configuration for the immediate future, with continued inspection, sampling and testing to serve as indicators for the safe operation of the dam.

Matilija Dam Environmental Restoration Feasibility Study
Structural Evaluation of Matilija Dam

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1.0 Project Authorization

Matilija Dam was constructed by County of Ventura in 1946-1947. Funding was provided by a County Bond, for the project purposes of recharging agricultural water aquifers and flood control. The reservoir was first completely filled to its 7,000 acre-feet capacity in 1952. The dam is currently owned by the County of Ventura, Public Works Agency, Flood Control Department and is operated by the Casitas Municipal Water District. Matilija Dam is a non-federal project that falls under the jurisdiction of the California Department of Water Resources, Division of Safety of Dams (DSOD).

2.0 Project Description (Structural)

2.1 General Description of Features

Matilija Dam is a variable radius, concrete thin arch dam located on Matilija Creek, a tributary to the Ventura River. The dam was designed by the Donald R. Warren Company Engineers and was constructed by Guy F. Atkinson Company, Bressie and Bevanda Construction Inc., and W.E. Kier Construction Company. Project features also include an overflow spillway, plunge pool, fish collection system, outlet works and water supply pipeline. Since original construction the arch has been modified twice as a result of adverse external and internal conditions caused by ASR and as recommended by Bechtel et. al.

2.2 Arch Dam and Thrust Blocks

The original construction configuration included a height of 195 feet (top of crest to foundation) and an arch crest length of 616 feet.[1] The arch was buttressed by right and left gravity thrust blocks and varied in thickness from 35 feet at the base to eight feet at the crown arch. (See Figures 2-1 and 2-2) The arches up to Elevation 1080 are filleted at the abutments. The dam crest as originally constructed was at Elevation 1138, except for the central overflow spillway section. The spillway varied in elevation from 1125 in the center portion to 1131 at the ends. The spillway runs along the crest for 535 of the 616-foot total arch length. A footbridge over the spillway allowed access from the left to right abutment. The arch has been modified twice over its life to its current configuration. (See Figure 2-3)

[1] The crest length to width ratio is approximately 3.2 based on original Crown Arch length of 616 feet (not including thrust blocks) and the Crown Cantilever of 194 feet, measured from foundation to top of arch crest at Sta. 4+93. The tallest cantilever in the current configuration is at Sta. 5+35 for a total height of 190 feet.

More than 60,000 cubic yard of concrete was used for dam construction (not including concrete used in the outlet works, grout and minor structures). The as-built plans indicate a minimum design compressive strength (f'_c) of 3,000 psi at 28-days and maximum size aggregate of 3-inches. The concrete was placed in five-foot lifts for the arches and thrust blocks. Vertical contraction joints are spaced every 40 feet, except for Blocks B, N, M and O, which are 45 feet wide and Block A which is 36 feet wide. Vertical contraction joints have a copper water strip near the upstream face and a galvanized steel water strip near the downstream face. All the contracting joints were keyed and pressure grouted. (See Figure 2-4) The contraction joints were built radial to the arch ring at elevation 960.

Matilija Dam is situated in a non-symmetric, wide-U shaped canyon. The streambed base is approximately 340 feet wide. The canyon is non-symmetric because the left canyon wall slopes at 31° from horizontal and the right canyon wall slopes at 61° from horizontal. Thus, the arch center points do not follow a linear reference line (shown as Axis B on the plans), but vary to fit the canyon. The total arch angle at crest is 120° 39' 49". The dam was designed with a left thrust block substantially larger than the right thrust block to compensate for the shallow left slope.

To minimize cantilever stresses in the relatively weak streambed foundation, a slip joint was constructed at elevation 960. Below Elevation 960 is a gravity foundation section of the dam. The slip joint was constructed by covering the gravity section with a graphite mortar and a 1/8-inch thick asbestos sheet.[2] The slip joint minimizes shear stresses acting on the foundation as a result of cantilever action but increases the arch stresses transmitted horizontally to the abutments.

2.3 Abutments and Foundation

The right abutment of the dam is composed of sandstone and shale interbeds, oriented approximately normal to the arch thrust of the dam. During construction, a three foot thick, weak shale bed was removed and backfilled with concrete. The left abutment of the dam is composed of fractured sandstone and shale beds, oriented acutely to the thrust of the arch dam. The abutments and foundation are considered weak materials that are not ideal for arch dam construction. Since 1965, the abutments have been monitored for deformation. The studies performed on the abutments have concluded that the abutment deformations have caused minor secondary effects on the dam structure. The abutments deflections are measured on a quarterly basis for signs of movement.

2.4 Plunge Pool, Concrete Apron and Fish Collection System

[2] The asbestos sheet is called out on Drawing S-10 of the as-builts as a "1/8 inch Johns Manville Service Sheet, lubricated face down."

Water flowing over the spillway falls into a plunge pool at the downstream toe of the dam. The toe of the dam is protected from scour by a six-foot thick concrete apron. The concrete apron extends approximately 75 feet downstream from the arch intrados (downstream face). The concrete apron and a training wall also slope up the left abutment for additional protection from spillway flows. The concrete apron is completely submerged, and forms the invert of the overflow plunge pool.

A reinforced concrete fish ladder rises from the downstream edge of the plunge pool. The fish ladder leads up the left abutment to a fish trap and holding tank. Originally, water flowed from the outlet works to the fish ladder, and cascaded into the plunge pool. Fish were collected in the trap, loaded into a truck and hauled to an area upstream of the dam. Over the years, the fish ladder has been damaged by debris falling over the spillway crest during high flows and remains inoperable.

2.5 Outlet Works

The original construction of Matilija Dam featured two outlet pipes: a 36-inch diameter outlet for the water supply pipeline and a 48-inch diameter outlet pipe for river discharge. Casitas Municipal Water District operates the outlet works controls. Currently, the 36-inch diameter outlet and a 42-inch outlet discharge up to 250 cfs. The 48-inch diameter outlet has been abandoned.

2.5.1 48-inch River Discharge Pipe

Since the 1960's the 48-inch diameter steel outlet pipe had sediment problems. The pipe was located near the center of the arch (Station 3+09) with an Inlet Elevation of 1000.80. Originally, the pipe had an upstream 48-inch sluice gate, which was replaced by a downstream 42-inch regulating valve. When the spillway was notched down to Elevation 1095 in 1965, a 20-foot wide by 8-foot high debris deflector was installed above the downstream outlet of the pipe. In 1969, a 20-foot high section was added to the intake riser to increase its top elevation to Elevation 1053. The sediment was noted on the plans to be at Elevation 1040. A discharge pipe was also added which used pressurized water or air to loosen up sediments, but never functioned adequately for pipe operation. The pipe was later abandoned to address dam safety and sediment issues.

2.5.2 36-inch Water Supply Discharge Pipe

The 36-inch outlet pipe is located at Station 1+25 and Elevation 1025. It was originally constructed with a 42-inch sluice gate on the upstream end of the pipe. An Outlet Gate House houses the inlet of the 36-inch outlet pipe. The outlet works have been modified to include an additional 42-inch diameter outlet pipe, a new intake structure and an additional 36-inch valve for river discharge. The maximum discharge rate is limited to 250 cfs because sediments and debris have clogged the intake structure screen baskets.

2.5.3 Valve House

Downstream of the dam, on the left abutment lies the Valve House. The control valves in the Valve House originally sent flow down the water supply line, into the fish ladder, or directly into the stream. The slope downstream of the Valve House is covered with riprap for erosion protection. Currently, all flow is discharged through an 18-inch control valve, directly into the river. The Ojai water supply line is no longer in use.

2.6 Instrumentation

The reader is encouraged to read the Bechtel report, "*Report on Review of Matilija Dam"* dated February 1965 and *"Review of Matilija Dam,"* dated August 1967 for an in depth review of project instrumentation. The system is briefly described here.

2.6.1 Survey Monitoring System

A survey control system was built into the original project for the purpose of detecting horizontal arch and abutment movements. Target points were set on the downstream face and access bridge of the dam and various control points located about the dam site.

Due to corrosion and notching the crest, almost all of the original survey markers had to be replaced in 1965. The new survey markers were monitored for two years and used to establish baseline conditions for further surveys.

In 1974, a new set of survey markers were established. The current survey control system is similar to the original system. Twelve survey markers are set on the downstream face and seven control points are set in pillars about the dam. Horizontal deformations are measured from a baseline established between two of the pillar control points. Upstream/downstream movement and left/right abutment movement are measured as perpendicular or parallel to the baseline. The 1974 Survey Markers are still monitored today.

Survey data from 1991 to 2001 was provided by the County of Ventura for use in this study. To date, a correlation between the current and old survey markers has not been identified.

2.6.2 Abutment Yield Measurement System

One of the recommendations of the 1964 Bechtel report was to continuously monitor the abutments for movement. The foundation and abutments were considered "weak" which raised concerns about the stability of the abutment rock, particularly the left abutment.

Eight deformation meters (gages) were installed in the foundation to monitor the overall modulus of deformation of the abutments. Four strain meters are located on each abutment and are installed generally in the direction of arch thrust. Two additional meters were installed in a direction radial to arch thrust to monitor slippage of the rock strata. The instruments are Carlson joint meters installed in three-inch diameter drill holes. The holes were drilled approximately 50 feet into the abutments. The meters read the change in length of rock between the dam abutment contact line and the bottom end of the drill hole. Readings are taken through use of a Carlson strain meter testing set. The elastic and inelastic deformations were established based on a cycling of reservoir operation and temperature change.

Abutment yield measurements were taken from 1965 to 1967 to establish baseline conditions of the abutment elastic and permanent inelastic deformations. The results are summarized in Bechtel 1967 Report. The abutment meter readings indicated negligible elastic and inelastic deformations and the movement are cyclical based on reservoir elevation when normalized for temperature. The report concluded that within the length of the meters, the abutment rock was adequately stable and the meters consistently reflect elastic behavior of the respective abutment.

The County of Ventura provided abutment meter data from 1991 to 2001 as part of this study. From 1991 to the present, abutment readings have been taken for six of the original eight meters. The lowest two meters were abandoned due to inundation at the downstream toe. The 1972 IECO study also included graphical plots of the abutment meter data from 1965 to 1972. Abutment meter data taken from 1991 to 2001 is discussed in Section 5.7 - Future Condition of Rock and Concrete Material Properties.

3.0 Previous Studies and Project Modifications

Shortly after construction engineers observed cracking on the downstream face of the dam. The recommendation was to monitor the cracking, however, the cracking worsened over time. In 1959, surveys of the crest survey plates indicated that the arch crown was moving in an upstream direction, against the loads imposed on the arch dam by the reservoir. The movement was thought to have been caused by internal expansion and cracking of the concrete. Since then, concrete mechanical properties have been evaluated approximately every ten years.

3.1 Bechtel Studies (1964 – 1967)

In 1964, the County of Ventura, Department of Public Works contracted with Bechtel Corporation of San Francisco to review Matilija Dam for the purpose of evaluating its safety. Bechtel conducted a two-phase study in which several concerns about the safety of the dam were raised. Bechtel's report made the following conclusions:

- All of the deterioration of concrete in the dam is due to reaction between the alkali in the cement and silica in the aggregate. The phenomenon was described

as *alkali-aggregate reaction* and is commonly known today as *alkali-silica reaction (ASR)*.
- The expansion, cracking and disintegration of the concrete was most severe in the upper 25 feet of the dam (above Elevation 1100). In this area, the factor of safety was well below the normally accepted minimum for arch dams, although the absolute value was indeterminate.
- Concrete core tests taken in this critically weakened zone (above Elevation 1100) indicate compressive strengths ranged from 1,200 to 2,600 psi. Portion of concrete encountered in the drilling disintegrated to such an extent that recovery of suitable test specimens was impossible and it should be assumed that the strength of the non-recovery concrete at this zones of the dam might be below 1,000 psi .
- Dam performance records (Bechtel 1965) indicated possibly serious instability of the abutments, particularly the left abutment. The foundation and abutment materials were weak and required monitoring to evaluate their integrity.

Bechtel's findings were also discussed with DSOD. In a letter to Bechtel and the County of Ventura dated March 19, 1965, DSOD expressed their concerns about the indeterminate safety factor of the dam and insisted upon several measures including monitoring, inspection and changes to reservoir operations.

Later that year, the County of Ventura decided to lower the stresses on the dam by "notching" or lowering the spillway crest, and to implement a surveillance program of instrumentation observation and measurement. The crest of the spillway was lowered from Elev. 1125 to 1095, between Station 1+75 and Station 4+55. The notching decreased the maximum reservoir level and thus decreased the loads and stresses acting on the dam. The surveillance program included (1) careful and frequent visual observations, (2) a new survey control system to monitor horizontal dam movement; and (3) meters in drilled holes at each abutment to monitor abutment elastic and inelastic deformations. The surveillance program continued for two years and the results were summarized the 1967 Bechtel report entitled, *"Review of Matilija Dam."*

The 1967 Bechtel report accomplished several tasks. They are summarized as follows:

- Modified Outlet Works. A hydraulic study was completed on the outlet works to allow greater controlled releases to the river. As a result, a new 36-inch valve was installed.
- Abutment meters: Readings of the abutment meters indicated that the abutment rock, neglecting small permanent inelastic deformations, was adequately stable within the lengths of the meters. The structural analysis noted that actual deformations were much less than deformations that would cause high stresses in the dam.
- Movement of the Dam. Since the old survey marker plates were not usable, new survey marker plates were installed. It was noted that prior to 1964, the crown

had moved upstream a total distance of 2.5 inches, however, the new survey markers plates had not been in place long enough to determine if the dam arch was still moving upstream due to reactivity or not. The structural analysis showed that the movement to date was acceptable, even when combined with the most unfavorable load combination of an empty reservoir with maximum temperature drop.

- Concrete Deterioration. Concrete cores were taken from several areas of the dam and tested to determine material properties and overall condition. It was determined that the concrete above Elevation 1095 would continue to deteriorate, however, continued deterioration of that section of the dam presented no hazard to the integrity of the dam itself. In addition, visual examination of the concrete below Elevation 1095 showed no evidence of concrete cracking, expansion, or deterioration. A non-destructive soniscope tests indicated the over-all state of the concrete to be "generally good" with however discrete areas of severely deteriorated concrete scattered through the upper portion of the dam
- Structural Analysis. A structural analysis was completed for the revised dam geometry (a.k.a. notching of the spillway was included). Assuming the most adverse load cases due to reservoir elevation, temperature change, silt load and seismic acceleration, the structural analysis showed that maximum stresses would not exceed the allow stress capacity of the concrete in the dam. The structural analysis also included the "wing walls" about the new spillway notch and the new footbridge added over the spillway notch.
- Future Observations and Testing. The report also stressed the importance of continuing the surveillance program and periodically testing the concrete for changes in material properties.

The 1967 Bechtel report concluded, *"Generally, the studies show that there is no reason to believe that the performance of the dam with respect to safety would be unsatisfactory in the foreseeable future."* The report recommended that the dam remain in service, monitoring be continued and the concrete be sampled and tested within five years.

In 1969 another modification was made to the project. Realizing that the sediments behind the dam would soon bury the 48-inch sluice gate, Ventura County added a 10-foot high riser to the sluice gate intake chamber. The plans noted that the sediment was approximately at Elevation 1040. The new riser extended the intake to Elevation 1053. The modification also included discharge pipes to dredge out the sediment about the intake openings.

3.2 International Engineers Company Studies (1972 – 1977)

In 1972 the County of Ventura contracted with International Engineers Company Inc., (IECO) to conduct a two-phase study on Matilija Dam. IECO completed the first report in August 1972, which was entitled, *"Matilija Dam Stress Investigations."* The report evaluated the stress conditions of the dam using a three-dimensional *finite element* *method* (FEM) of structural analysis. The study determined that the governing load case was the combined static and dynamic load case. The report made several recommendations including:

- Re-establishing a survey control system
- Increased monitoring of the instrumentation
- Replacement of the footbridge spanning the spillway
- Sampling and testing of the concrete and foundation rock

The second report, prepared in 1975, investigated the actual strength properties of the concrete and rock foundation, and compared the results with assumptions from the previous study. The investigation included the vertical core samplings from the crest to the rock foundation at three locations and six horizontal samplings cored through the face of the dam. BTC Laboratories provided testing services for this phase of the study and Woodward-Clyde Consultants performed the in situ shear wave velocity measurements. The results were summarized in the December 1975 IECO report entitled, *"Matilija Dam, Phase II Investigation, Determination of In Situ Concrete and Foundation Properties,"* and are summarized as follows and tabulated in Table 3.1:

- Visual inspections, concrete core sampling and testing suggested that the upper 40 feet of the dam, Elevation 1090 and above, was in an advanced state of deterioration. The main indicators of this condition were the low values of static and dynamic elastic moduli, and the extent of pattern cracking. Below Elevation 1090 the concrete quality increases and concrete is in better condition with fewer cracks and higher strengths. The compressive strengths for vertical core samples ranged from 3,450 to 7,850 psi below Elevation 1090. The compressive strengths for horizontal core samples ranged from 5,620 to 7,000 psi below Elevation 1090.
- The concrete test results show that the actual static elastic moduli were higher than the static elastic moduli assumed in the 1972 IECO stress analysis. The dynamic elastic moduli are lower than assumed in the 1972 stress analysis; however, the adjustment was not anticipated to have a significant impact on the results.
- Foundation is generally weak—it was described as highly fractured, locally weathered and faulted which resulted in very low core recovery. Although the actual foundation elastic moduli are higher than the values assumed in the 1972 stress analysis, the low core recovery makes the laboratory results more optimistic than actual conditions.
- Petrographic and gel fluorescence examination show that the aggregates are in an advanced state of alkali-silica reactivity. Thus, the concrete will continue to deteriorate.
- Based on the findings of this report it was determined that the 1972 IECO stress analysis results were still valid.

		TABLE 3-1:CONCRETE CORE TEST DATA Matilija Dam Ventura County, California				
Sample No.	**Year**	**Core Orientation**	**Plan Location**	**Elevation (feet)**	**Compressive Strength (psi)**	**Bulk Specific Gravity**
P128164	1947	NA	Apron	NA	3950	NA
P128166	1947	NA	Apron	NA	3875	NA
P128168	1947	NA	Arch	1110-1115	5085	NA
P128170	1947	NA	Arch	NA	4555	NA
P128172	1947	NA	Arch	1100-1105	5085	NA
P128342	1947	NA	Arch	1110-1115	4645	NA
A:1.7-2.4 ft	1964	Vertical	*CL of Arch, Sta 0-0.5	1136	2300	2.26
B:9.6-10.4 ft	1964	Vertical	*CL of Arch, Sta 1+49	1118	5000	NA
B:15.7 ft	1964	Vertical	*CL of Arch, Sta 1+49	1113	NA	2.24
B:16.0-16.8 ft	1964	Vertical	*CL of Arch, Sta 1+49	1112	2500	2.32
C:6.8-7.6 ft	1964	Vertical	*CL of Arch, Sta 2+35	1118	3700	2.32
C:14.0-14.8 ft	1964	Vertical	*CL of Arch, Sta 2+35	1111	4300	2.25
D:3.2-4.0 ft	1964	Vertical	*CL of Arch, Sta 3+22	1122	4900	2.31
D:8.3-9.1 ft	1964	Vertical	*CL of Arch, Sta 3+22	1117	5100	2.32
D:14.1-14.9ft	1964	Vertical	*CL of Arch, Sta 3+22	1111	3900	2.43
D:22.2-23.0 ft	1964	Vertical	*CL of Arch, Sta 3+22	1103	2600	NA
D:26.0-26.9 ft	1964	Vertical	*CL of Arch, Sta 3+22	1099	4000	NA
E:4.7-5.5 ft	1964	Vertical	*CL of Arch, Sta 4+30	1120	5600	2.33
E:9.0-9.8 ft	1964	Vertical	*CL of Arch, Sta 4+30	1116	1200	2.33
E:13.0-13.8 ft	1964	Vertical	*CL of Arch, Sta 4+30	1112	5300	2.31
E:19.2-20.0 ft	1964	Vertical	*CL of Arch, Sta 4+30	1106	5000	2.26
F:7.0-8.0 ft	1964	Normal to DS Face	7.5 ft from DS Face of Arch, Sta. 5+45	976	3500	2.37
F:8.0-9.0 ft	1964	Normal to DS Face	8.5 ft from DS Face of Arch, Sta. 5+45	975	2600**	2.33
1	1975	Normal to US Face	2 ft from US Face of Arch, Sta. 5+25	1067	7000	2.34
2	1975	Normal to US Face	2 ft from US Face of Arch, Sta. 3+40	1067	5945	2.37
3	1975	Normal to US Face	2 ft from US Face of Arch, Sta. 1+90	1067	5622	2.37
4	1975	Normal to DS Face	2 ft from DS Face of Arch, Sta. 5+25	977	6637	NA
5	1975	Normal to DS Face	2 ft from DS Face of Arch, Sta. 3+40	977	5593	2.36
6	1975	Normal to DS Face	2 ft from DS Face of Arch, Sta. 1+90	977	5951	NA
A2	1975	Vertical	CL of Arch, Sta 4+78	1089	2505	NA
A3	1975	Vertical	CL of Arch, Sta 4+78	1078	5726	2.28
A4	1975	Vertical	CL of Arch, Sta 4+78	1058	4928	2.31
A5	1975	Vertical	CL of Arch, Sta 4+78	1048	6138	2.38
A6	1975	Vertical	CL of Arch, Sta 4+78	1038	7098	2.34
A7	1975	Vertical	CL of Arch, Sta 4+78	1018	7537	NA
A8	1975	Vertical	CL of Arch, Sta 4+78	996	7728	2.34
A9	1975	Vertical	CL of Arch, Sta 4+78	978	6388	2.3

A10	1975	Vertical	CL of Arch, Sta 4+78	958	5732	2.25
B1	1975	Vertical	*CL of Arch, Sta 1+60	1123	4384	2.35
B3	1975	Vertical	CL of Arch, Sta 1+60	1074	NA	NA
B4	1975	Vertical	CL of Arch, Sta 1+60	1062	7158	2.31
B5	1975	Vertical	CL of Arch, Sta 1+60	1049	6530	2.35
B6	1975	Vertical	CL of Arch, Sta 1+60	1037	7850	NA
C3	1975	Vertical	CL of Arch, Sta 1+35	1086	4842	2.35
C4	1975	Vertical	CL of Arch, Sta 1+35	1072	5115	2.36
1A	1996	Normal to DS Face	1 ft from DS Face of Arch, Sta 5+10	1068	NA	NA
1B	1996	Normal to DS Face	2 ft from DS Face of Arch, Sta 5+10	1068	2690	2.37
2D	1996	Normal to DS Face	3 ft from DS Face of Arch, Sta 3+40	1068	NA	NA
2E	1996	Normal to DS Face	4 ft from DS Face of Arch, Sta 3+40	1068	2890	2.39
3C.2	1996	Normal to DS Face	3 ft from DS Face of Arch, Sta 1+87	1068	NA	NA
3C.3	1996	Normal to DS Face	3 ft from DS Face of Arch, Sta 1+87	1068	5670	2.4
4A	1996	Normal to DS Face	1 ft from DS Face of Arch, Sta 5+25	982	6610	2.35
4C.2	1996	Normal to DS Face	3 ft from DS Face of Arch, Sta 5+25	982	NA	NA
5B.2	1996	Normal to DS Face	3 ft from DS Face of Arch, Sta 3+50	982	NA	NA
5C.2	1996	Normal to DS Face	5 ft from DS Face of Arch, Sta 3+50	982	5470	2.38
6C.2	1996	Normal to DS Face	3 ft from DS Face of Arch, Sta 2+36	982	5420	2.38
6E.3	1996	Normal to DS Face	5 ft from DS Face of Arch, Sta 2+36	982	NA	NA
1A	1997	Normal to US Face	1 ft from US Face of Arch, Sta 5+10	1068	4214	2.34
1C	1997	Normal to US Face	3 ft from US Face of Arch, Sta 5+10	1068	NA	NA
1D	1997	Normal to US Face	5 ft from US Face of Arch, Sta 5+10	1068	2885	2.36
2A	1997	Normal to US Face	1 ft from US Face of Arch, Sta 3+40	1068	5290	2.38
2D	1997	Normal to US Face	4 ft from US Face of Arch, Sta 3+40	1068	NA	NA
2E.2	1997	Normal to US Face	6 ft from US Face of Arch, Sta 3+40	1068	3060	2.38
3A	1997	Normal to US Face	1 ft from US Face of Arch, Sta 1+89	1068	6130	2.4
3C	1997	Normal to US Face	4 ft from US Face of Arch, Sta 1+89	1068	NA	NA
3E	1997	Normal to US Face	6 ft from US Face of Arch, Sta 1+89	1068	4590	2.39

Notes/Explanation:

All compressive strength data from 1947 are from quality control cylinders of concrete cured and tested at 28-days.
* This section of the dam has been removed.
** Sample F: 8.0 to 9.0 feet was obtained at a lift joint, which probably explains the low compressive strength result.
Therefore, this sample was not included in the statistical average.
CL = center line
DS = downstream
US = upstream
NA = not available or not applicable
psi = pounds per square inch
Extract from Harza Consulting Engineers & Sciencetists;Report of Stress Analysis Matilija Dam, March 1999.

Following the 1972 IECO report, a series of modifications were made to the spillway and outlet works of Matilija Dam. The changes are summarized as follows:

- Monitoring Facilities (1974): new survey markers and pillars replaced old survey control system.
- New Outlet Pipe (early 1970's): a new 42-inch outlet pipe was added to the Outlet Works at Station 1+25, Elevation 1040.5. The pipe discharges directly to the river.
- Widened Spillway Notch (1977): removed the footbridge, piers and parts of arch dam to widened spillway notch to the current configuration. The sluice gate operators and platform from the bridge were relocated to Elev. 1110 and access provided.
- Control House and Electrical (1979): relocated the electrical panels and conduits, from the control house on the left thrust block to a new control house.
- Intake Structure (1980): Replaced the sheetpile cofferdam and trash rack about the 36-inch and 42-inch outlet pipes with a new intake structure. The new intake structure was built of reinforced concrete. Basket screens and stop logs from Elevation 1064 to 1086.5 prevent sediments from entering the intake structure.

3.3 Division of Safety of Dam Studies (1979)

While the changes were in progress at Matilija Dam, DSOD made a visual inspection of the project as part of the *National Dam Inspection Act*, Public Law 92-367. In 1979 DSOD in conjunction with federal authorities prepared a report entitled, *"National Dam Inspection Report, Phase I."* The report was the first phase of the nation-wide program to perform a preliminary review of the safety of the dam. DSOD performed the visual inspection of the dam and a review of existing records.

DSOD updated the hydraulic flood routing for the dam and concluded that the widened spillway crest could safely pass the PMF (76,108 cfs) regardless of reservoir operation method. DSOD studied the regional seismicity and noted that the seismic values were higher than previous studies. DSOD reported that the concrete continued to deteriorate about the area of the spillway notch and that the reservoir was gradually filling in with sediments. They predicted that the reservoir would not be completely filled until sometime after the year 2000. In summary, DSOD concluded that the 1972 and 1975 IECO studies were still valid and that the dam was safe for use currently and in the near future.

No information was made available by the County of Ventura for any dam modifications that took place after 1979. It is assumed that the 48-inch sluice gate was abandoned sometime in the 1980's. The County of Ventura is also looking for survey records that correlate previous survey data with the survey data from 1965 to 1972 and 1991 to 2001.

No other modifications or studies pertaining to the structures were provided for use in this study.

3.4 Harza Studies (1996 – 1999)

The most current review on the structural adequacy of Matilija Dam took place between 1996 and 1999. The County of Ventura contracted with Harza Consulting Engineers and Scientists, who drafted a report entitled, *"Report of Stress Analysis, Matilija Dam."* This report summarized all the previous studies and compiled all the concrete data, revised the concrete material properties, and applied updated structural loads to the 1975 IECO stress analysis. BTC Laboratories Inc. performed sampling and testing of the concrete in 1996 and 1997.

The concrete sampling and testing revealed that the concrete continues to deteriorate due to ASR. The average concrete compressive strengths for specific drill holes were lower than cores taken in 1975 at similar locations, particularly in the upper arches of the dam. The deterioration was spreading to a wider area than shown in 1975 however, the overall quality of the concrete was deemed comparable to the values used in the 1972 IECO stress analysis.

Harza then updated the loading conditions to reflect current silt levels, probable maximum flood elevations and maximum credible earthquake peak horizontal ground accelerations. The updated loads were then applied as correction factors to the 1972 IECO stress analysis and updated stresses obtained. The methodology used to update the stress analysis was reviewed by DSOD and found to be a reasonable approach, according to Harza report.

The 1999 Harza report concluded that, *"based on the data and engineering analyses documented in this report, the existing configuration of Matilija Dam may be considered stable under the updated static and dynamic loading conditions defined herein."* The report also recommended that additional concrete sampling and testing be performed eight to ten years from the time of the report. A full-scale finite element method analysis was advised depending upon the extent of change of concrete properties.

3.5 Dam Removal Demonstration Project

The Matilija Dam Removal Demonstration Project was a project granted by the National Fish and Wildlife Foundation and in conjunction with other local interests. The purpose of the project was to remove an upper-most section of the dam to test methods for future removal of the entire dam. The project explored four removal methods: diamond-wire cutting, hydraulic splitting, expansive grouting, and pneumatic chipping. Diamond-wire cutting and pneumatic chipping proved to be the best suited removal methods for Matilija Dam. Interestingly enough, previously undocumented steel reinforcement was encountered during the removal process. The reinforcement was though to have been sacrificial formwork support steel used in the original construction of the ogee crest.

4.0 Project Current and Future Use

4.1 Water Storage

The use of Matilija Dam has changed significantly from its original purposes of water storage and flood control. The water stored at Matilija Dam is currently discharged to the river rather than used for recharging groundwater in the Ojai area. The Casitas Municipal Water District (CMWD) operates the project to maximize diversions taken at the Robles Diversion Dam while maintaining the minimum required flows down the Ventura River. The water discharged out of Matilija Dam is collected at Robles Diversion Dam and transported to Lake Casitas via the Robles-Casitas Diversion Channel.

4.2 Flood Control

Matilija Dam is no longer operated for flood control purposes. The spillway notching and sediment accumulation have eliminated the original flood control benefits provided by the project. A Federal Emergency Management Agency (FEMA) approved 100-year flood plain has been delineated from Matilija Dam to the ocean.

4.3 Future Use

The sediment deposits and spillway crest lowering have reduced the reservoir storage capacity from the original 7,000 acre-feet to less than 500 acre-feet. The reservoir is expected to fill in with sediments up to the spillway crest by the year 2020, based on average annual sediment inflows. Although flow is currently being retained, and discharged directly to the river, the reservoir could be used for limited water supply for the remainder of its useful life. Matilija Reservoir has also been used as a source of water by fire fighting helicopters. CMWD also may utilize the reservoir as an emergency water supply if ever required.

5.0 Current and Future Condition

5.1 Alkali-Silica Reaction

The affects of ASR are well documented at Matilija Dam. ASR is a chemical reaction that occurs between the alkalis from the cement (and other sources), certain siliceous constituents present in the aggregate and moisture (present in concrete or environment). The three components (alkali, siliceous materials and moisture) have to be present for ASR to occur.

5.2 Concrete Aggregate Sources

Bechtel (1965) states that the fine aggregate for Matilija Dam construction was obtained from sources on the Santa Clara River near Saticoy, approximately a 25 mile haul to the project site. Available sources do not identify the reason for selection of particular borrow sites but a potential reason is that the designers believed the material at the site to be of poor quality.[3] Concrete mix design number (47-41) supplied by Smith Emery Company, dated February 13, 1947 indicated that Saticoy Rock Company in Saticoy supplied the fine aggregate. It was also indicated in the mix design that Rock Product Company in Irwindale California supplied the coarse aggregate. A copy of the mix design for one concrete cylinder break is appended to "Phase I Inspection Report for Matilija Dam", prepared by The State of California, The Resources Agency, Department of Water Resources, Division of Safety of Dams, dated June 1979. Saticoy Rock Company and Southern Pacific Milling Company in Oxnard California have operated aggregate pits in the Santa Clara River for more than 60. Test results on aggregates from the Santa Clara River documented by U.S. Army Corps of Engineers (Technical Manual 6-370) dating from February 1957 indicate that aggregates from the Santa Clara River are to be judged borderline to potentially reactive when tested in accordance to CRD-C 128. Test results dating from February 1948 till present indicate aggregates from San Gabriel River formation to be non-reactive (USACE TM 6-370). The source is approximately 75 miles from the project site.

5.3 Cement Sources

Previous reports did not provide information on Portland cement source used for concrete construction. However, the information furnished in the concrete mix design number (47-41) provided the maximum size aggregate to be 1.5-inches minus, five sacks of cement per cubic yard and a water cement ratio of 0.787. Type II cement with 0.3 Type I pozzolanic admixture were recommended. The specified compressive strength was 3,000 psi. at 28 day. Prior to construction it was known that the fine aggregate was potentially reactive and the specifications required the use of low alkali cement (limiting the alkali content to 0.6 percent). There are no cement quality records to indicate the actual percent alkalis in the cement used in the concrete placements. Bechtel (1965) indicates:

"A major question in this specific case seems to be the actual alkali content of the cement as supplied and its variability. Although there were reasonable assurances that the alkali content was generally below that required in the specifications, it can not be assumed

[3] In a conversation with Mr. Sergio Vargas, CVFCD, on May 15, 2002, he mentioned that the contractor may have also obtained fine aggregates from a local source. The as-built drawings show concrete placed prior to April 17, 1947, which was thought by Mr. Vargas to indicate concrete made with the local fine aggregates for the first 30 – 40 feet of several of the blocks.

with certainty to have been so at all times during construction. In particular, there are no mill certification records available for the month of August 1947, a time when most of the presently severely deteriorated lifts were placed."

5.4 Compressive Strength

The 1999 Harza report summarized the change in compressive strengths of the concrete over time for holes 1 through 6. The Harza report indicates that the compressive strengths have peaked in some locations, but not in all locations of the dam. For the stress analyses, Harza concluded that compressive strength of 4,500 psi could be used, just as it was in the 1975 IECO study. Dynamic compressive strengths were assumed to be 25% greater than the static compressive strengths.

5.5 Tensile Strength

Tensile strengths in the 1975 IECO report were found to be approximately 10 percent of the compressive strength. There was not a correlation between tensile strength and location within the dam, however, the larger diameter core samples were thought to have been more representative than the 2.5-inch smaller diameter samples. The average tensile strength of the 1975, 5.9-inch diameter samples was 595 psi.

Harza stated that tensile strengths normally range between 7 and 11 percent of the compressive strength. The 1996 and 1997 test results fall within that range. Harza concluded that a tensile strength of 475 psi was acceptable for design criteria. Dynamic tensile strengths were 25% greater than the static tensile strengths.

5.6 Other Concrete and Rock Material Properties

The concrete and rock material properties used in the 1972 IECO stress analysis are summarized in Table 5-1. Concrete material properties from the 1975 IECO and 1996 Harza investigations confirmed the 1972 design assumptions.

Table 5-1 Concrete and Rock Material Properties

Material Property	1972 IECO	1975 IECO	1999 Harza
unit weight of water	62.5 pcf	NT	62.5 pcf
unit weight of Concrete	145 pcf	145 pcf [1]	145 pcf [1]
equivalent hydrostatic pressure of slit	20 pcf	NT	20 pcf
Poisson's ratio of concrete	0.2	0.02-0.34	0.23
Poisson's ratio of rock	0.25	NT	NT
coefficient of thermal expansion of concrete	5.6 $\times 10^{-6}$ per °F	NT	NT
Static modulus of elasticity of all concrete	2.0 x 10^6 psi	3.29 x 10^6 psi	2.35 x 10^6 psi
Dynamic modulus of elasticity of intact concrete	3.0 x 10^6 psi	0.54-2.32 x 10^6 psi	NT
Static modulus of elasticity of deteriorated concrete	25,000 psi	2.78 x 10^6 psi [2]	NT
Dynamic of elasticity of deteriorated concrete	25,000 psi	540,000 psi	NT
modulus of elasticity of foundation	0.5-1.5 x 10^6 psi [3]	2.6 x 10^6 psi [2]	NT
Dynamic modulus of elasticity of foundation	0.5-1.5 x 10^6 psi	1. 14 x 10^6 psi	NT
chemical expansion coefficient for upper left abutment	0.0009	NT	NT
chemical expansion coefficient for upper right abutment	0.0005	NT	NT

Notes:
[1] based on 1972 specific gravity of 2.34 and a 1996 specific gravity of 2.35
[2] only one complete test was due to fractured core recovery and the value is not a representative of the section
[3] modulus of elasticity of foundation listed here are within the value report by Bechtel in 1965 as cited in Appendix C, Geotechnical Report.
NT not tested

5.7 Future Condition of Rock and Concrete Material Properties

The various material testing reports indicate that the ASR will continue to cause cracking, expansion and deterioration of the concrete for the remaining life of the project (Bechtel et al). Over time the reactivity rate in the upper arches of the dam may diminish as all the alkalis in the cement have reacted. However, alkali from the external environment could further react with the silica in the aggregate thus continuing the process indefinitely. Furthermore, as concrete expands in the upper arches, confining pressures will be released and the ASR will become more prevalent in the mid and lower sections of the dam. This will be visually evident as the cracking and expansion will propagate from the areas about the notched spillway to the surrounding areas of low confinement stress and eventually to all areas of the dam.

The difficulty in estimated the probable life of the structure stems from the fact that there is no way to accurately forecast the reduction in mechanical properties over time due to ASR. However, there are no formal rules for defining strength variation over time in the concrete industry. The situation is further complicated at Matilija Dam since the rate of expansion varies across different sections of the dam based on confining stresses, exposure to moisture and temperature. Thus, it is not possible to accurately predict the future concrete material properties for Matilija Dam. Concrete sampling and testing in ten to twelve years should give a better indication of the concrete material life cycle.

Rock material properties are assumed not to vary significantly over time. The strain meter survey data from 1991 to 2001 shows predominantly cyclical behavior with maximum deflections less than one inch in compression. The two strain meters monitoring slippage of the rock strata show maximum expansion less than one-tenth of an inch and insignificant creep. Minimum and maximum deformations, measured from the original 1965 positions, between August 1991 and December 2001 are summarized in Table 5-2.

Table 5-2 - Abutment Deformations
August 1991 to December 2001

Drill Hole #	Maximum (in)	Minimum (in)
1L	-0.9989	-0.9261
2L	-0.1174	-0.0110
2AL	0.0888	0.0724
3L	*	*
1R	-0.0356	-0.0037
2R **	-0.0178	-0.0070
2AR	0.0126	0.0089
3R	*	*

Notes:
(-) negative for compression
(+) positive for expansion
* = drill hole abandoned
** = The 12/27/2001 reading of DH-2R shows tension deformation of 0.0135 inches.

While the meters have shown predominately cyclical behavior, abutment meter D.H.-1L experienced some increasing compression deformation starting in May 1967 and progressing to some time between June 1972 and August 1991. The June 20, 1972 reading showed only (–0.177 inch) of compression, whereas, the data between August 1991 and Jan 2001 shows cyclical action ranging between (-0.9261) and (-0.9989 inch) of compression. The abutment seems to have stabilized, probably due to the 1977 spillway widening project which removed an area of concrete near the left abutment severely deteriorated by the ASR.

Overall, the results are well within historical ranges and exhibit similar behavior patterns to the 1967 Bechtel and 1972 IECO studies. Thus, the abutments do not exhibit significant changes from the earlier studies and it can be assumed at this time that the existing information on rock material properties reflects current conditions. Note that detailed geological investigations, reflecting state of the art methods of evaluation, may be required in the PED phase of this environmental restoration project.

6.0 Structural Evaluation

For the purpose of establishing baseline conditions in this feasibility study, the results of previous studies will be evaluated against current Corps of Engineers criteria for arch dams. Current Corps of Engineers guidance is found in the Engineering Manual entitled, *EM1110-2-2201 - "Arch Dam Design,"* dated May 31, 1994. This criteria is based on the state of the art practice for arch dam design. Since most of the study alternatives call for the removal of Matilija Dam, a detailed structural analysis of the dam will not be completed as part of the evaluation of various alternatives. However, if the preferred alternative recommends that the arch dam, or a significant portion of the arch dam, remain in place, then a complete structural analysis using the criteria specified in EM 1110-2-2201 should be completed. This evaluate of previous studies does not preclude the need for satisfying current arch dam design criteria and a new structural analysis should be performed in order to make that determination.

6.1 Load Combinations

Arch dams are designed for a variety of loading conditions and load combinations. Load cases are usually unique for each dam, depending on site-specific conditions. General load cases outlined in EM 1110-2-2201 are restated in the following Table 6-1. Site-specific load cases for Matilija Dam would be derived from the General Load Cases. For example, Load Cases SUN3 and DUN2 newly constructed dams and would not be applied to Matilija Dam. The load combinations identified in the 1999 Harza and 1972 IECO analyses addressed all of the critical load combinations for Matilija Dam, including the affects of chemical expansion due to ASR.

Table 6-1 - Load Combinations from EM 1110-2-2201

Static Load Combinations	
SU1	Minimum usual concrete temperature. Reservoir elevation occurring at that time. Dead Load. *usual load condition*
SU2	Maximum usual concrete temperature. Reservoir elevation occurring at that time. Dead Load. *usual load condition*
SU3	Normal Operating Reservoir Condition. Concrete temperature occurring at that time. Dead Load. *usual load condition*
SUN1	Reservoir at spillway crest elevation. Concrete temperature at that time. Dead Load. *unusual load condition*
SUN2	Minimum design reservoir elevation. Concrete temperature occurring at that time. Dead Load. *unusual load condition*
SUN3	End of construction condition. Structure completed, empty reservoir. Temperature Load. *unusual load condition*
SE1	Reservoir at Probable Maximum Flood (PMF) elevation. Concrete temperature occurring at that time. Dead Load. *extreme load condition*
Dynamic Load Combinations	
DUN1	Operating Basis Earthquake (OBE) plus static load case SU3. *unusual load condition*
DUN2	OBE plus static load case SUN3. *unusual load condition*
DE1	Maximum Design Earthquake (MDE) plus static load case SU3. *extreme load condition*
Notes: 1) Load combinations are categorized according to *loading conditions* of *usual*, *unusual* or *extreme*. Different factors of safety are applied to each loading condition, per EM1110-2-2201, Chapter 11.	

6.2 Design Criteria

The stresses resulting from the static and dynamic load combinations in arch dam design are compared against structural design criteria to ensure that adequate factors of safety are achieved. Chapter 11 of EM 11102-2201 specifies design criteria for static and dynamic load cases, according to the *usual*, *unusual* or *extreme* loading conditions, see Table 6-2.

Table 6-2 - Criteria for Static and Dynamic Load Conditions

Design Criteria		Static			Dynamic	
		Usual	Unusual	Extreme	Unusual	Extreme
Allowable Compressive Stress	f_c =	$f'_c / 4$	$f'_c / 2.5$	$f'_c / 1.5$	$f'_{cd} / 2.5$	$f'_{cd} / 1.5$
Allowable Tensile Stress	f_t =	f'_t	f'_t	f'_t	f'_{td}	f'_{td}
Factor of Safety against sliding	FS_s =	2.0	1.3	1.1	1.3	1.1

Notes:
f'_c = compressive stress
f'_{cd} = dynamic compressive stress
f'_t = tensile stress
f'_{td} = dynamic tensile stress

EM 1110-2-2201 notes that the acceptable performance of arch dams under dynamic load cases is a complicated process that requires additional evaluation beyond the design criteria given in the table. Rather, the allowable values are only the first step in determining the safety criteria and should not be regarded as absolute limits.

The sliding factors of safety are based are based on comprehensive geological field investigations and evaluation. Foundation investigations are essential for arch dams because of the critical relationship between the dam and the foundation. Rock material properties play an important role in the FEM analysis.

Although the load combinations performed in the analyses for Matilija Dam were not categorized according to the Corps of Engineers *load conditions*, the analyses did compare the allowable stresses with calculated stresses to determine if the dam met practical factors of safety at that time.

6.3 Past Structural Analyses

Past structural analyses have been performed for Matilija Dam to analyze the safety of the dam due to adverse external and internal conditions, and to design project modifications as a result of those adverse conditions. While the concrete properties have degraded over time due to the ASR, the loading has increased substantially. The loading from previous structural analyses on Matilija Dam is summarized in Table 6-3. This section also includes a brief description of the loads and how they are applied.

6.3.1 Dead Loads

Dead loads are due to the weight of the dam and appurtenants structures. The unit weight of concrete used in the studies for Matilija Dam was 145 pcf, based on concrete testing.

Table 6-3 – Past Structural Analysis; Loading Conditions

	Silt	Hydraulic	Seismic	Notes
1965 Bechtel	Silt Elev.: 1037 (present) Silt Elev.: 1069 (future) Silt EHP: ~20 pcf	PMF WSE: 1138 PMF: 70,000 cfs Normal WSE: 1125 Min WSE: 1069 Spillway Crest: 1125	HGA: 0.10g (pseudu-ostatic only)	• Analysis software: SADSAM • Governing load case: empty reservoir & max temp drop (construction load) • Original geometry
1967 Bechtel	Silt Elev: 1069 (future)	PMF Elev.: 1113.7 PMF: 70,000 cfs Normal WSE: 1095 Spillway Crest: 1095	HGA: 0.10g (pseudo-static only)	• Modified geometry (notched spillway) • Analyzed individual arches & cantilevers only
1972 IECO	Silt Elev.: 1040 (present) Silt Elev.: 1069 (future est.) Silt EHP: 20 pcf	PMF Elev.: 1113.7 PMF: 70,000 cfs Normal WSE: 1095 Spillway Crest: 1095	PHGA: 0.35g (San Andreas) PHGA: 0.45g (Santa Ynez)	• Finite element model • Analysis software: 3D-SAP • Includes chemical expansion
1979 DSOD	N/A	PMF Elev.: 1111.0 PMF: 76,108 cfs Normal WSE: 1095 Spillway Crest: 1095	MCE: 0.7g PHGA (Santa Ynez)	• Phase 1 study, concludes 1972 stress analysis still valid
1997 Harza	Silt Elev.: 1090 (present) Silt Elev.: 1095 (future est.) Silt EHP: 20 pcf	PMF Elev.: 1111.0 PMF: 76,108 cfs Normal WSE: 1095 Spillway Crest: 1095	MCE: 0.7g PHGA (Santa Ynez)	• Corrective Factors to 1972 FEM • Updated material properties
Current (2002)	Silt Elev.: 1075 (present) Silt Elev. 1095 (future est.) Silt EHP: 20 pcf	PMF Elev.: (not estimated) PMF: 70,000 (prelim. estimate) Normal WSE: 1095 Spillway Crest: 1095	MCE: 0.77g PHGA (Mission Ridge-Arroyo Parida-Santa Ana) MDE: 0.77g EPGA OBE: 0.34g EPGA	• Consistent with 1999 Harza study

Notes:
Silt EHP: Equivalent Hydraulic Pressure of Sediments acting on upstream face of dam.
PMF: Probable Maximum Flood
WSE: Water Surface Elevation
PHGA: Peak Horizontal Ground Acceleration
EPGA: Estimated Peak Ground Acceleration (horizontal)
Santa-Ana Fault is a fault group consisting of the Mission Ridge-Arroyo Parida-Santa Ana Faults. Santa Ynez Fault is a south dipping, high angle, reverse fault

6.3.2 Hydraulic Loads

Hydraulic loads are a function of reservoir operation. Various load combinations, with water at minimal, normal, and probable maximum reservoir elevations, are calculated to represent the different operation conditions of the dam. The force of water acts horizontally against the arch dam and increases as function of depth squared. The unit weight of water used in the studies for Matilija Dam was 62.4 pcf.

The operation of Matilija Dam has changed over time due to the notching of the spillway, accumulation of sediment, varying estimates of the probable maximum flood, and other factors. Thus, the hydraulic loads acting on the dam have changed, from study to study. The USACE Los Angeles District (LAD) Hydrology and Hydraulics Section made a preliminary estimate of the PMF that shows a PMF peak inflow in the range of the 1979 DSOD PMF peak inflow. Since the operation of the dam is not expected to change, the hydraulic loads are expected to remain the same in the future.

6.3.3 Silt Loads

The sediment that has accumulated in the reservoir has caused an increase of the horizontal forces acting against the upstream face of the dam. The silt load increases as a function of the depth squared and equivalent hydrostatic fluid pressure of the sediment. The USACE (LAD) Geology and Investigation Section estimates the equivalent hydrostatic fluid pressure to be approximately 20 psf. This same value was used in past analyses for Matilija Dam. The silt load is expected to increase as the amount of sediment is expected to increase until the sediments reach the spillway crest at Elevation 1095. The 1999 Harza analysis estimated the sediment elevation to be between 1085 and 1090 and used 1090 in the analysis. Investigations for this study estimate the current sediment elevation to be approximately 1075.

6.3.4 Temperature Loads

Temperature loads result from the differences between the closure (grouting) temperature and concrete temperatures in the dam during its operation. The *closure temperature* is the concrete temperature at the time of grouting of the contraction joints. The closure temperature is one of the most important construction parameters in arch dams because once the monolith joints are grouted, the structure is assumed to become monolithic and the arch action begins. Future concrete temperatures are compared to the closure temperature to determine if the concrete is contracting or expanding, and therefore, causing tensile or compressive stresses in the arches.

Load combinations from the 1965 Bechtel and 1972 IECO studies included maximum temperature drop in conjunction with a full reservoir and silt loading. The 1972 IECO study concluded that maximum temperature drop would increase the stresses by 15% to 20% for certain regions of the dam, but that maximum temperature drop was not a governing load condition. The concrete temperatures variations stated in the 1965 Bechtel study are assumed to be the actual design values. The 1972 IECO study corrected the temperatures for thickness of the concrete section. Otherwise, the temperatures used in the maximum temperature drop analyses have not changed, nor are expected to change over time.

6.3.5 Earthquake Loads

USACE criteria specifies two levels of design earthquakes. These are the Operational Basis Earthquake (OBE) and the Maximum Design Earthquake (MDE). OBE is defined as a ground motion having a 50 percent chance of exceedance in 100 years. The dam is expected to respond elastically under the OBE (assuming continuous monolithic action along the entire length of the dam). MDE is the maximum level of ground motion for which the arch dam should be analyzed, and it is usually equated to the maximum credible earthquake (MCE). MCE is defined as the largest reasonable possible earthquake that could occur along a recognized fault or within a particular seismic source. Under the MDE, the dam is allowed to respond nonlinearly and incur significant damage, but without a catastrophic failure in terms of loss of life or economics.

As noted in Table 6-3, the seismic loads at Matilija Dam have increased substantially from the original design. The 1964 Bechtel study applied a pseudo-static acceleration of 0.1 times gravity for horizontal seismic loads. The 1972 IECO study used a dynamic FEM and time histories for two separate maximum credible earthquakes, including vertical accelerations as 60% of the horizontal accelerations. This analysis found that the seismic load case, in combination with static loads, was the governing load case.

The 1999 Harza applied correction factors to the 1972 IECO study and concluded that despite instantaneous tensile stresses in excess of concrete tensile capacity, that overall, the dam was stable under the seismic loads. This conclusion is concurrent with Corps criteria for MCE's, which allows nonlinear and inelastic behavior as long as catastrophic failure does not occur. An OBE has not been evaluated for Matilija Dam. Seismic loads at the site are not expected to increase.

6.3.6 Chemical Expansion Loads

Similar to the way temperature affects tensile and compressive stresses through expansion and contraction, the chemical expansion increases the stresses acting within the concrete as a result of ASR.

The 1972 IECO study included an analysis of stresses due to chemical expansion. The chemical expansion was applied in the FEM as equivalent coefficients of thermal expansion. In their study, IECO noted that the actual rate of chemical expansion on the dam was unknown. In order to determining the rates of chemical expansion, IECO performed an analysis of several steps. First, IECO analyzed individual loads to confirm the assumed elastic properties of the concrete and rock foundation. Second, IECO analyzed the structure's response to determine the theoretical chemical expansion. They found that the theoretical chemical expansion was not realistic because the analysis did not account for stress relief as a result of cracking and the actual measured deflections overstated the theoretical expansion rate because of the open cracks. Based on this data, IECO made a qualitative estimate and used effective rates of expansion in the analysis. Additional tensile stresses were then determined and included in applicable load combinations.

Chemical expansion loads were not evaluated in the 1999 Harza study. In order to evaluate current rates of chemical expansion, a comprehensive study, similar to the one conducted by IECO in 1972, would have be implemented. Generally, the loads due to chemical expansion will continue to act upon the structure, cause stress relief through cracking, and detract from the structure's overall ability to resist the water, silt and seismic loads.

6.3.7 Summary of Previous Studies

The maximum compressive and tensile stresses resulting from previous studies are summarized in Table 6-4. The stresses in bold italic text are calculated stresses that exceed current Corps of Engineers criteria as defined in Table 6-2 and based on the concrete properties of the 1999 Harza study.

For dynamic load cases, the dynamic compressive and tensile stresses reflect concrete material properties that occur under rapid loading conditions. Testing for dynamic compressive and tensile stresses has not been performed specifically for Matilija Dam. Studies on ratios of dynamic to static strengths for various mass concrete mixes show a range of 0.73 to 1.36 for compressive tests and 0.98 to 1.73 for tensile tests. The 1999 Harza study increased the allowable stresses by 25% for the dynamic load cases, which is an increase common in most codes and guidance for dynamic load cases in concrete design.

The stresses for Study 5 (from the 1972 IECO analysis, see Table 6-4) exceed the Corps current criteria for allowable compressive and tensile stresses. The loads from Study 5 included a normally full reservoir, with silt loads and assumed effective chemical expansion. Since that time, the spillway has been widened and most of the deteriorated concrete removed. Thus, the stresses do not portray current conditions. The 1999 Harza study indicates that the deterioration continues to spread. Test results for the upper forty feet of the dam (Elevation 1088 and above) showed a decrease in compressive strength of approximately 13%. In order to estimate current loads caused by chemical expansion, an evaluation of current effective expansion rates would need to be made.

Table 6-4 Summary of Maximum Stresses versus USACE Allowable Stresses

Study	Year	Load Case	Reservoir Elev.	Silt Elev. Against Dam	Max Temp Drop	Chemical Exp.	PHGA	USACE Loading condition	Max Comp. Stress	Max Tensile Stress	USACE Allow. Comp. Stress	USACE Allow Tensile Stress
Bechtel	1967	Combination A Flood & Temperature	1113.7		Yes	No		*extreme*	740 psi	-120 psi	3000 psi	-475 psi
Bechtel	1967	Combination B Flood & Temp. & Silt	1113.7	1069	Yes	No		*extreme*	960 psi	-130 psi	3000 psi	-475 psi
Bechtel	1967	Combination C Water, Temp Silt and EQ	1095	1069	Yes	No	0.10g	*extreme*	960 psi	-130 psi	3750 psi	-600 psi
ICEO	1972	Study 1 - Full Reservoir & Exist. Silt Level	1095	1040	No	No		*usual*	732 psi	-51 psi	1125 psi	-475 psi
ICEO	1972	Study 2 - Max Reservoir & Exist. Silt Level	1113.7	1040	No	No		*extreme*	855 psi	-110 psi	3000 psi	-475 psi
ICEO	1972	Study 3 - Full Reservoir, Exist. Silt Level with Max Temp Drop	1095	1040	Yes	No		*unusual*	844 psi	-189 psi	1800 psi	-475 psi
ICEO	1972	Study 4 - Inc. Stresses for Chemical Expansion[1]	n/a	n/a	No	Yes		*n/a*	1183 psi	-635 psi		
ICEO	1972	Study 5 - Full Reservoir & Silt w/ Assumed Effective Chemical Expansion	1095	1040	No	Yes		*usual*	***1223 psi***	***-591 psi***	1125 psi	-475 psi
ICEO	1972	Study 6A - Incremental Stress for Future Silt Load[1]	n/a	1040 to 1069	No	No		*n/a*	+67 psi	-17 psi		
ICEO	1972	Study 6B - Incremental Stress for Possible Abutment Deformation[1]	n/a	1040 to 1069	No	No		*n/a*	+127 psi	-70 psi		
ICEO	1972	EQ Study 1 - San Andreas Fault Richter Magnitude 8+[1]	n/a	n/a	No	No	0.35g	*n/a*	871 psi	-1002 psi		
ICEO	1972	EQ Study 2 - Santa Ynez Fault Richter Magnitude 6.5 to 7[1]	n/a	n/a	No	No	0.45g	*n/a*	619 psi	-712 psi		
ICEO	1972	Combined Static Plus Dynamic EQ1	1095	1040	No	No	0.35g	*extreme*	1323 psi	***-681 psi***	3750 psi	-600 psi
Harza	1999	Static Stress Analysis Results with PMF Loading	1111	1090	No	No		*extreme*	1460 psi	-288 psi	3000 psi	-475 psi
Harza	1999	Dynamic Stress Analysis Results	n/a	n/a	No	No	0.70g	*n/a*	490 psi	-1110 psi		
Harza	1999	Combination of Static & Dynamic Stresses	1095	1090	No	No	0.70g	*extreme*	1460 psi	***-930 psi***	3750 psi	-600 psi

Note:
Values shown in ***bold italics*** exceed current Corps of Engineers criteria for arch dams.
[1] Analysis for incremental or dynamic stresses to be combined with other stresses.

The most critical load cases from past studies appear to be for the seismic MCE load cases. Both the 1972 IECO and the 1999 Harza studies determined that a Combined Static and Dynamic Load case had tensile stresses higher than the allowed tensile capacities. As noted previously, nonlinear, inelastic behavior is allowed for the MDE as long as a catastrophic failure does not occur. As pointed out in the studies the instantaneous tensile stresses will be relieved by slight, momentary opening and closing of cracks and joints that will not result in a catastrophic failure of the dam. Thus, the results are consistent with USACE criteria for MDE's.

Although an OBE has not been evaluated for Matilija Dam, the PHGA's used in the 1972 IECO study are approximately the same as the OBE PHGA identified by the USACE as part of this feasibility study. Applying the *unusual* load condition criteria to the IECO seismic study shows that some areas of the dam have instantaneous tensile stresses (-681 psi) that exceed the allowable dynamic tensile strength (-600 psi). USACE criteria specifies that for the OBE and normal operating reservoir conditions, the stresses in the dam must remain totally within the elastic range of concrete, to assume that the dam behaves as a monolithic structure. This criteria has not been met based on the previous studies, however, it is important to note that the OBE is a *serviceability* requirement to ensure that dam remain completely operable following OBE level earthquakes.[4] Since there is no inherent risk of dam failure, or potential for property damage or lose of life should the dam become inoperable, the OBE criteria may not be applicable without conducting a comprehensive risk assessment study. At a minimum, a FEM should be completed for the OBE, with the most current project geometry and material properties, if this feasibility study determines that the dam should remain in service.

6.4 Hazard Classification

6.4.1 USACE Hazard Potential Classification System

The Hazard Potential Classification System adopted by the Corps of Engineers (ER 1110-2-1155) classifies dams based the functional integrity of the project rather than the structural integrity of individual project features or components. The losses are classified in four general categories: Loss of Life, Property Losses, Lifeline Losses and Environmental Losses. Dams are rated as *low*, *significant*, or *high* hazard based on the proximity of the population at risk, and the impact upon life and property due to the loss of essential services.

The hazard potential classification for Matilija Dam is based on a review of FEMA flood inundation maps for failure of Matilija Dam and current orthographic photos (2001) of the area downstream of Matilija Dam. The inundation maps were prepared in 1973 by the County of Ventura. The reader should note that the current conditions at the dam

[4] Note that the 1979 DSOD study stated that no matter how the dam is operated, the project will safety pass the PMF.

would probably yield a smaller flood wave than the flood wave developed in the 1973 study. Also note that this classification does not attempt to evaluate the structural integrity of Matilija Dam, nor estimate the probability of dam failure. This classification does not imply that there are deficiencies with Matilija Dam that render it unsafe, and as far as the author is concerned, the risk of failure is no different than that of other dams of similar age, type and condition. A proper risk assessment study and updated inundation mapping should be completed in order to determine actual probabilities of failure and potential consequences. That said, in the remote event of a dam failure, Matilija Dam is considered a High Hazard Dam based on the almost certain loss of life, the disruption of critical facilities and access, major damages to public and private property, and extensive mitigation required for environmental damages.

6.4.2 Potential Loss of Life

Loss of Life is classified as *high*, *significant* or *low* based on the certainty that one or more lives will be lost due to project failure or incorrect operation of the project. The certainty is based on the population at risk, proximity to the project, the flood wave travel time, the warning time, availability of access and other factors.

If Matilija Dam were to fail unexpectedly, the potential for loss of life is almost certain. The closest occupied buildings are less than 1,000 feet downstream of the dam in Matilija Hot Springs. The travel time is less than five minutes for the peak flood wave. The flood wave would reach Matilija Hot Springs without warning. The only access out of Matilija Hot Springs is down an access road in the canyon, which will also be inundated. The peak flood wave will reach the confluence with the east branch of the Ventura River and inundate a stretch of State Route 33(five minutes to peak). Downstream of that there are several orchards with habitable structures just north of the Mira Monte City Boundary, located near the Robles Diversion Dam (nine minutes to peak). Larger population centers within the 100-year flood plain and most likely within the inundation area of a dam failure are in Live Oaks Acres (28 minutes to peak), Oak View, Casitas Springs (48 minutes to peak) and other downstream communities. Downstream of Matilija Hot Springs, the loss of life is probable due to the short flood wave times, limited warning times and somewhat limited access out of inundated areas.

6.4.3 Property Losses

Property losses are classified as either: *direct economic losses* due to flood damaged homes, businesses, and infrastructure; or *indirect economic losses* due to the interruption of services provided by either the failed facility or by damaged property or infrastructure downstream. If Matilija Dam were to fail unexpectedly, flooding would damage a significant number of private and public structures. The large volume of sediment behind Matilija Dam would produce extensive mudflows causing permanent streambed alterations and heavy flood damage. Several orchards located within the flooded area would be destroyed. The repair and clean up of the massive amounts of sediments would have enormous economic impacts to those communities and would take a long time to reestablish.

Flooding would also have indirect economic impacts downstream. Matilija Dam supplies a small amount of water, approximately 1,000 acre-feet annually, to a local water purveyor. This water supply would be lost if the dam failed. The Robles Diversion Dam would be filled in with sediment and most likely overtopped and breached. The Robles Diversion Dam supplies water to Casitas Dam, a major water supply dam for Ventura County. The communities of Mira Monte, Live Oaks Acres, Oak View, Casitas Springs would have significant damage to their infrastructure including roads, utilities, and emergency services. U.S. Highway 101, and State Routes 33 and 180 would most likely be inundated and possibly partially destroyed. Two existing rail lines may also be affected. Thus, major indirect impacts would occur and would take several years to replace.

6.4.4 Lifeline Losses

Lifeline Losses are classified according to amount of disruption to critical or essential facilities and access. Disruption of essential lifeline services or access to these services during or following a catastrophic event can result in indirect threats to life. The flood wave resulting from a failure of Matilija Dam would disrupt essential services including the water provided by the Robles Diversion Dam, damages to highways and roads within the canyon, and the inundation of hospitals and other emergency facilities in the communities of Oak View, Casitas Springs and other downstream communities.

6.4.5 Environmental Losses

Environmental losses are evaluated as the amount of mitigation required to correct the damages caused by a dam failure. The environmental losses consider the incremental damages that occur between the project failure flood wave and the maximum flood wave damages expected without existence of project. Since Matilija Dam presently has no real flood storage capacity, the inundated area remains the same whether the dam is present or not. However, the large amount of sediment behind the dam would cause extensive damages to the environment in the inundated area that would not normally occur if the dam did not exist. Damage would include streambed alterations, habitat loss, and wildlife loss. Thus, extensive mitigation would be required to recover from the environmental losses due to a dam failure.

7.0 Conclusions

Arch dams have been recognized over the centuries for their extraordinary strength. Some of the oldest masonry structures still standing in the Middle East are arch dams. Current records show that there has never been a structural failure of an arch dam due to an earthquake (EM 1110-2-2201, page 11-4). In contrast, some modern dams (constructed in the 1900's) would probably show signs of distress if analyzed using state-of-the-art methods, even though some of these dams have experienced severe earthquakes without suffering any structural damages.

In the case of Matilija Dam, the most current structural analyses and material testing have indicated that the dam is stable, even if strict Corps of Engineers design criteria are not completely achieved. In fact, the results of the analyses and material testing suggest that Matilija Dam could be expected to remain in its current configuration, without extensive modification, for the next fifty years. This conclusion is based on the following findings:

- The County of Ventura provided the Corps of Engineers with sufficient amounts of information pertaining to the safety of the dam. Structural studies and concrete sampling and testing have been performed approximately every ten years with the most recent structural analysis performed less than three years ago. DSOD dam safety inspections are conducted annually. Thus, existing information is current and provides a good history of the project.

- ASR has and will continue to deteriorate the concrete in the dam. Thus, the concrete material properties are expected to degrade over time and as noted in previous studies, concrete sampling and testing is recommended by the year 2008.

- Instrumentation data provided for this study indicates that the abutments and arch movement are within tolerable limits. The movement is predominantly cyclical, acting as a function of reservoir elevation and temperature. The current program of instrumentation monitoring should continue.

- The operation of the dam has been adversely impacted by the spillway notching and sedimentation of the reservoir, to the extent that the project is no longer used for the original purposes of groundwater recharge or flood control. Several modifications to the outlet works have been made in order to maintain controlled discharge capabilities. The reservoir capacity has been reduced from 7,000 acre-feet to less than 500 acre-feet. The future operation of the project appears to be extremely limited.

- Past structural studies and modifications have ensured that the project remain in a safe and stable condition despite an overall increase of loads. Silt loading is expected to increase until the reservoir is filled to spillway elevation. Earthquake loads have increased from the original design, but are not expected to increase in the future. Although estimates for the PMF have increased since original design, flood loads have been reduced as a result of spillway notching. Loads due to temperature are not expected to change. Lastly, stresses due to chemical expansion loads will increase and spread to other parts of the dam, but be relieved through cracking and expansion of the concrete. Future modifications might be necessary to remove deteriorated concrete and to reduce loading, however, it is not possible to predict the future reduction of material properties using current concrete technology.

- Should this feasibility study determine that Matilija Dam remain in service, several studies are recommended within the next ten years. First, the concrete should be sampled and tested for dynamic material properties. Second, an updated FEM should be performed, using state-of-the-art technology with the most current material properties and dam geometry incorporated into the model. Third, a comprehensive risk assessment study should be performed to aid the dam owner in making decisions about future safe operation of the dam.

- Should this feasibility study determine that Matilija Dam should not remain in service, the concrete section could be removed and processed as recycled aggregates. The recycled aggregate could be used for road base, riprap or miscellaneous fill (depending on product size). The use of the recycled aggregate for concrete is not recommended due to ASR and the low values of the bulk specific gravity. Local commercial aggregate suppliers may be interested in hauling and processing any removed concrete at minimal costs.

8.0 References

8.1 American Concrete Institute, ACI Committee 201, *"ACI 201.2R-92 - Guide to Durable Concrete,"* Section 5.1.1, ACI, Detroit Michigan, 1992.

8.2 Bechtel Corporation, *"Preliminary Report on Review of Matilija Dam,"* prepared for the County of Ventura, Department of Public Works, October 1964.

8.3 Bechtel Corporation, *"Review of Matilija Dam,"* prepared for the County of Ventura, Department of Public Works, February 1965.

8.4 Bechtel Corporation, *"Review of Matilija Dam,"* prepared for the County of Ventura, Department of Public Works, August 1967.

8.5 FEMA, *"Flood Inundation Map of Matilija Dam Failure,"* Ventura County Sheet 1 thru 6, August 1973.

8.6 Harza Consulting Engineers and Scientists, *"DRAFT Report of Stress Analysis, Matilija Dam,"* prepared for the County of Ventura, Department of Public Works, 1 March 1999.

8.7 International Engineering Company, Inc., *"Matilija Dam, Stress Analysis,"* prepared for the County of Ventura, Department of Public Works, August 1972.

8.8 International Engineering Company, Inc., *"Matilija Dam Phase II Investigation, Determination of In Situ Concrete and Foundation Properties,"* prepared for the County of Ventura, Department of Public Works, December 1975.

8.9 State of California, Department of Water Resources, Division of Safety of Dams, *"Phase I Inspection Report for Matilija Dam,"* prepared for USACE, Sacramento District, June 1979.

8.10 Letter from Mr. Robert B. Jansen, Supervisor, Safety of Dams, California Department of Water Resources, to Mr. Karl Taylor, Assistant Manager, Bechtel Corporation, Subject: Matilija Dam , No. 86, dated 19 March 1965.

8.11 USACE, Sacramento Area Flood Control Agency, State of California Reclamation Board, *"American River Watershed, California, Long-Term Study, Final Supplemental Plan Formulation Report / Environmental Impact Statement / Environmental Impact Report,"* Volume III, Appendix C, February 2002.

8.12 USACE, ER 1110-2-1155, *"Dam Safety Assurance Program,"* 12 September 1997.

8.13 USACE, EM 1110-2-2201, *"Arch Dam Design,"* page 11-4, 31 May 1994

8.14 Ventura County Flood Control District, *"Matilija Dam Removal Demonstration Project 00-351, Final Report,"* prepared for National Fish and Wildlife Foundation, August 2001.

8.15 Ventura County Flood Control District, *"Matilija Dam,"* select record drawings, 6 February 1948.

8.16 Ventura County Flood Control District, *"Matilija Dam Riser Extension,"* Record Drawings dated 16 September 1969.

8.17 Ventura County Flood Control District, *"Matilija Dam Modification, Monitoring Facilities, Phase II,"* Record Drawings dated 27 August 1974.

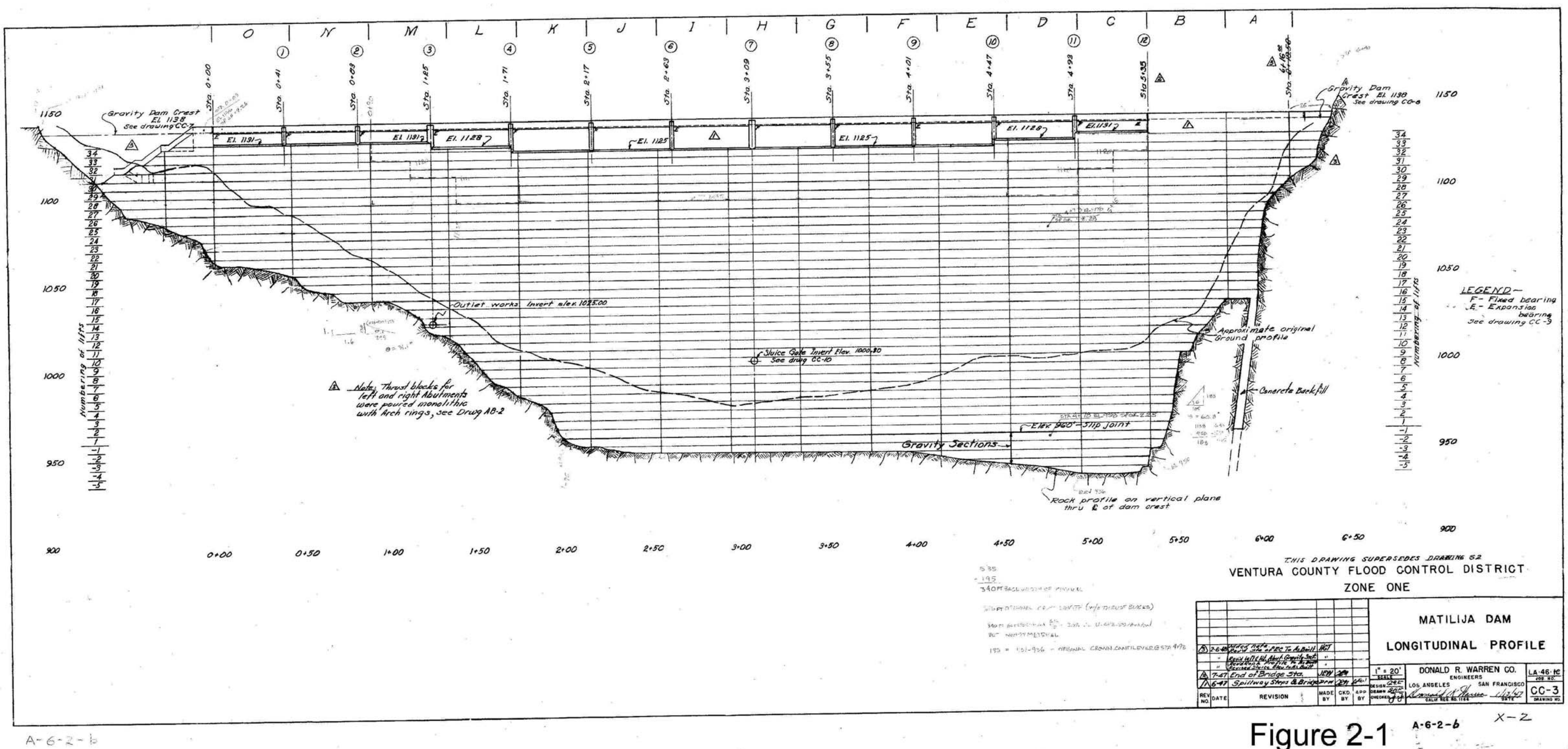
8.18 Ventura County Flood Control District, *"Matilija Dam Modification – Phase II,"* Record Drawings dated 3 August 1977.

8.19 Ventura County Flood Control District, *"Matilija Dam Modification – Phase II, Control House and Electrical,"* Record Drawings approved 20 June 1979.

8.20 Ventura County Flood Control District, *"Matilija Dam Modification – Phase III, Intake Structure,"* As-Built Plans approved 31 December 1080.

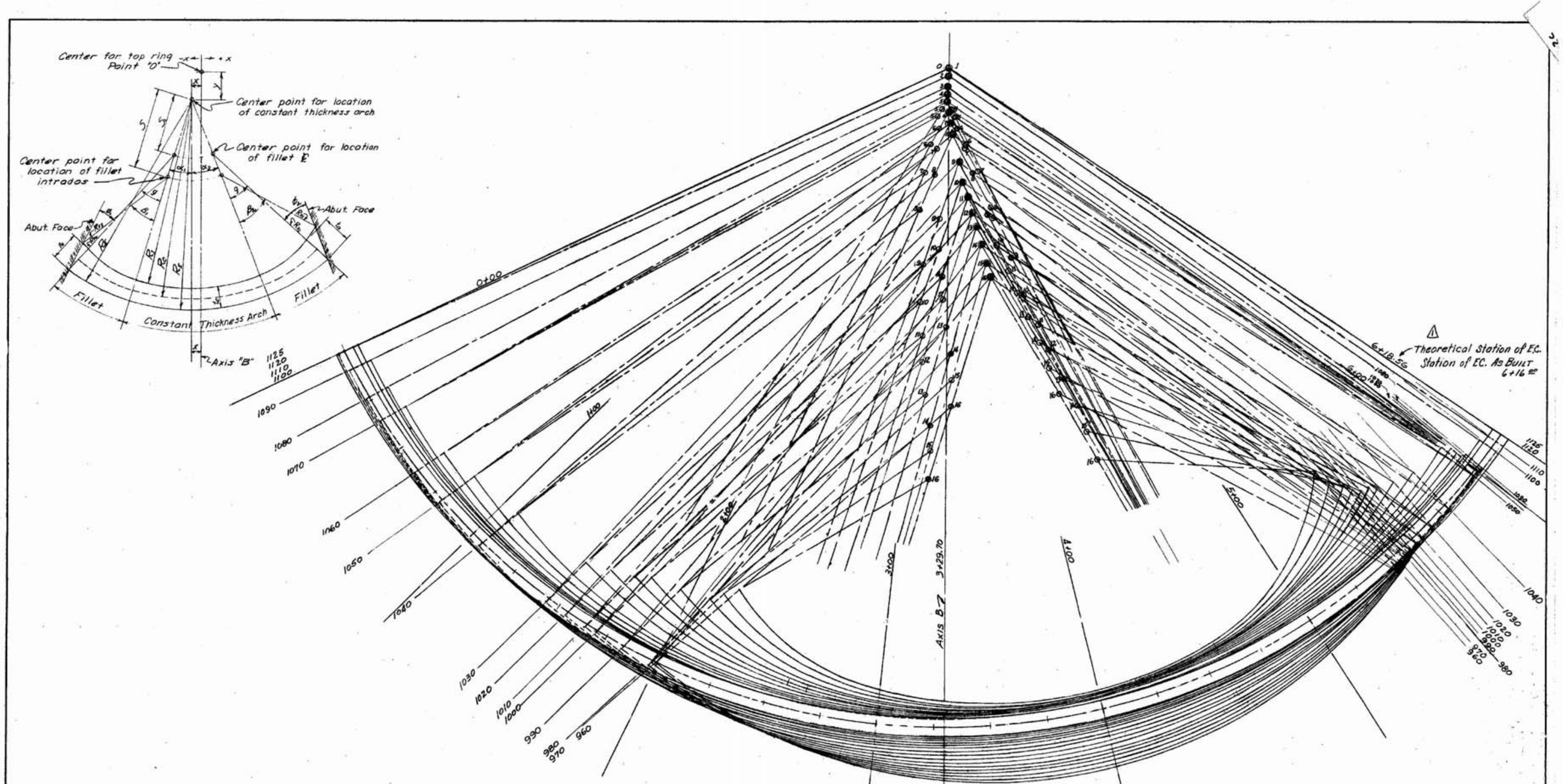
8.21 Ventura County Flood Control District, *"Matilija Dam Evaluation Demonstration"* Record Drawings approved 21 June 2001.

8.22 Ventura County Flood Control District, *"Matilija Dam, Horizontal Deformation & Abutment Yield Measurements – August 1991 thru December 2001,"* 31 December 2001.



Gravity Dam Crest EL 1138 See drawing CC-7
Gravity Dam Crest EL 1138 See drawing CC-8
El. 1131
El. 1131
El. 1128
El. 1125
El. 1125
El. 1128
El. 1131
Outlet works Invert elev. 1025.00
Sluice Gate Invert Elev. 1000.30 See drwg CC-10
Note: Thrust blocks for left and right Abutments were poured monolithic with Arch rings, see Drwg AB-2
Elev 960 - Slip Joint
Gravity Sections
Rock profile on vertical plane thru ℄ of dam crest
Approximate original Ground profile
Concrete Backfill
Numbering of lifts
LEGEND -
F - Fixed bearing
E - Expansion bearing
See drawing CC-9
THIS DRAWING SUPERSEDES DRAWING 62
VENTURA COUNTY FLOOD CONTROL DISTRICT
ZONE ONE
MATILIJA DAM
LONGITUDINAL PROFILE
DONALD R. WARREN CO. ENGINEERS
LOS ANGELES SAN FRANCISCO
1" = 20'
LA-46-1C
CC-3
7-47 End of Bridge Sta.
6-47 Spillway Steps & Bridge
REVISION
A-6-2-b
X-Z

Figure 2-1



⚠ Note:- For As Built Abutment thickness see drwg AB-2. See Drwgs CC-15 & CC-16 For Denotion for O, A & B Blocks.

Arch at Elevation	Center for Extrados at Point	Coordinates x (ft)	Coordinates y (ft)	Angles for Constant Thickness Arch α_1	Angles for Constant Thickness Arch α_2	Fillet Angle g	Fillet Angle Left Ab. β_1	Fillet Angle Right Ab. β_2	Distance C to C Intrados S	Distance C to C ℄ S_1	Radii for Constant Thickness Arch Extrados R_e	Radii for Constant Thickness Arch ℄ R_c	Radii for Constant Thickness Arch Intrados R_i	Radii for Fillet Arch ℄ R_{fc}	Radii for Fillet Arch Intrados R_{fi}	Thickness Crown t_c	Thickness Abut t_a	Abut. Angle Left Ab. ϕ_1	Abut. Angle Right Ab. ϕ_2	Total Extrados Angle
Crest 1125	0	0.00	0.00	64 34 56	56 04 53	0° 0' 0"	0 0 0	0° 0' 0"			296.5	292.5	288.5			8.0	8.0		0° 00	120 39 49 ⚠
Above 1120	0	0.00	0.00								296.5	292.5	288.5			8.0	8.0			
1120	0	0.00	0.00		56 32 13						296.5	292.5	288.5			8.0	8.0			
1110	1	0.00	0.00	64 37 08	55 18 42						296.5	292.5	288.5			8.0	8.0			120 25 50
1100	2	-0.08	3.67	65 26 38	55 52 48						293.38	288.88	284.38			9.0	9.0		3° 00	121 19 20
1090	3	-0.25	8.62	64 25 24	54 35 48						290.17	285.17	280.17			10.0	10.0		4° 30	119 01 12
1080	4	-0.33	11.50	61 51 23	54 54 41	0° 0' 0"	0° 0' 0"	0° 0' 0"			288.17	282.67	277.17			11.0	11.0		5° 00	116 46 04
1070	5	-0.42	15.00	31 23 50	26 27 50	29 19 03	29 43 38	29° 49'	7.50	3.95	285.50	279.50	273.50	275.55	265.69	12.0	13.0		6° 00	115 43 20
1060	6	0.00	19.58	27 40 07	28 03 43	28 40 52	29 32 38	30 01	16.30	8.38	282.60	275.60	268.60	267.22	252.30	14.0	16.0		8 00	111 27 40
1050	7	0.58	24.17	25 30 08	29 00 06	28 31 03	29 57 18	30 32	24.73	12.92	279.43	271.43	263.43	258.51	238.69	16.0	19.0		8° 00	109 00 30
1040	8	1.96	29.67	23 10 58	27 37 05	27 14 21	29 13 59	30 17	36.14	19.31	275.50	266.50	257.50	247.19	221.36	18.0	22.0		10° 45	101 36 06
1030	9	4.83	41.78	18 47 25	28 43 10	26 18 32	29 07 38	30 51	48.52	26.68	265.35	255.35	245.35	228.67	196.83	20.0	25.0		14° 30	95 01 10
1020	10	6.36	50.75	18 15 49	28 49 48	26 43 22	30 17 24	32 42	56.63	31.80	258.10	247.10	236.10	215.30	179.47	22.0	28.0		14 30	94 11 14
1010	11	8.58	57.08	17 39 00	28 57 04	27 06 59	31 28 58	34 34	64.45	36.95	253.08	241.08	229.08	204.13	164.63	24.0	31.0		15 00	93 12 08
1000	12	10.50	64.09	17 56 17	29 06 02	27 56 14	33 05 57	36 53	69.68	40.62	247.00	234.00	221.00	193.38	151.32	26.0	34.0		16 00	94 04 18
990	13	12.33	70.33	16 09 11	29 52 27	28 11 30	34 23 00	39 22	77.47	46.26	242.18	228.18	214.18	181.92	136.71	28.0	37.0		16 45	92 03 16
980	14	14.41	78.05	14 52 46	30 47 31	28 46 18	36 05 10	43 42	83.18	50.76	236.25	221.00	205.75	170.24	122.57	30.5	40.5		18 45	91 20 34
970	15	17.00	86.67	15 59 28	30 49 59	30 15 42	38 58 57	48 09	87.10	54.19	229.00	212.75	196.50	158.56	109.39	32.5	44.0		20 30	93 38 54
960	16	18.60	92.90	16 41 11	30 53 25	31 50 36	47 32 51	56 06	93.31	59.55	224.20	206.70	189.20	147.15	95.88	35.0	48.5		21 00	95 09 12

STATE OF CALIFORNIA
DEPARTMENT OF PUBLIC WORKS
DIVISION OF WATER RESOURCES
APPLICATION NO. 86
APPROVED AS TO SAFETY
SEPT. 16, 1949
DATE
STATE ENGINEER

Figure 2-2

VENTURA COUNTY FLOOD CONTROL DISTRICT
ZONE ONE

MATILIJA DAM
GEOMETRIC LAYOUT OF ARCHES

⚠ Rev'd E.C. To As Built & Added Note — HGT

DATE — APPROVED BY V.C.F.C.D. ZONE 1 — REV. NO. — DATE — REVISION — MADE BY — CKD. BY — APP. BY

1" = 20'

DONALD R. WARREN CO.
ENGINEERS
LOS ANGELES — SAN FRANCISCO
LA46-10
CC-4

A-6-2-C

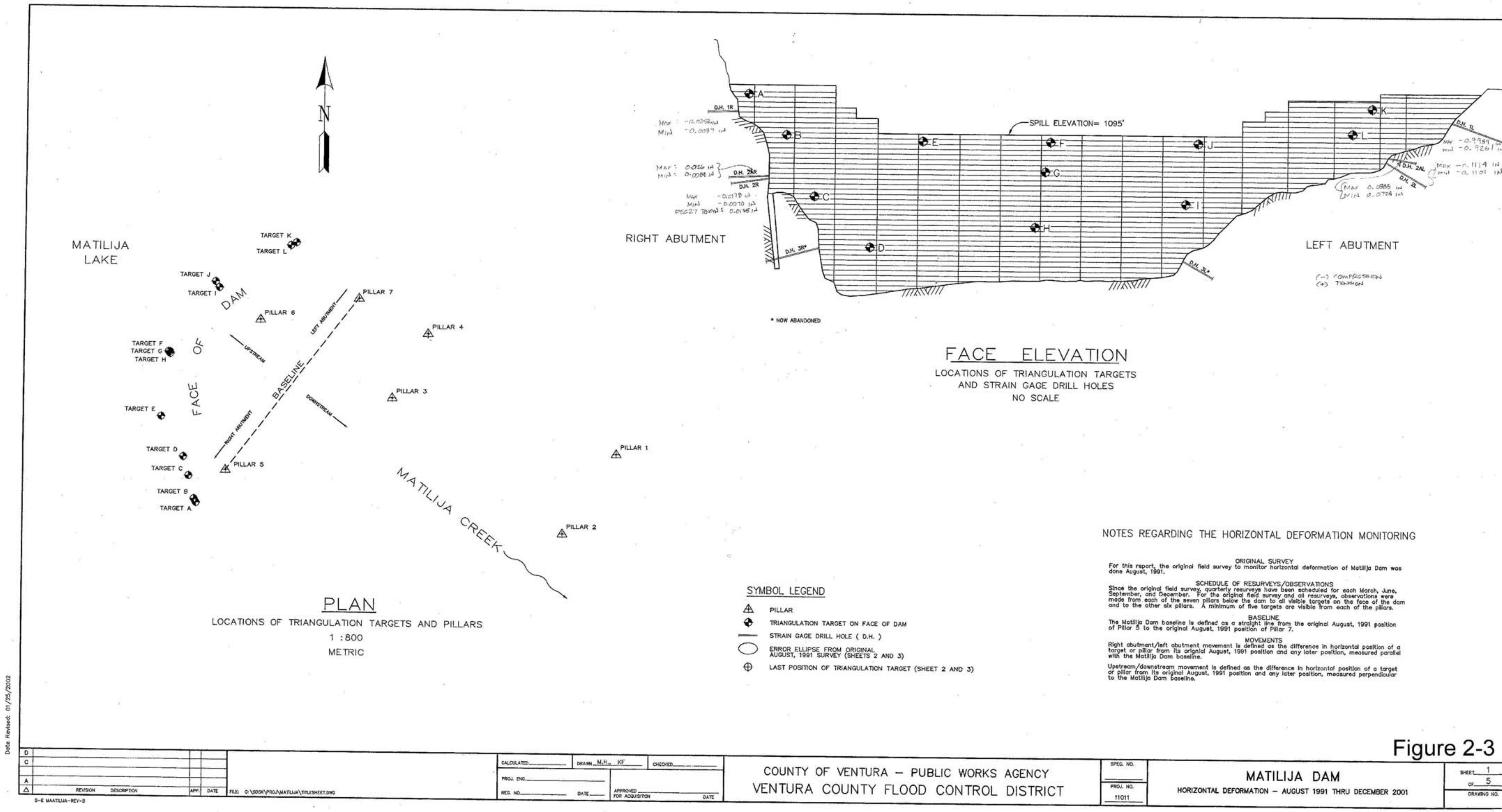
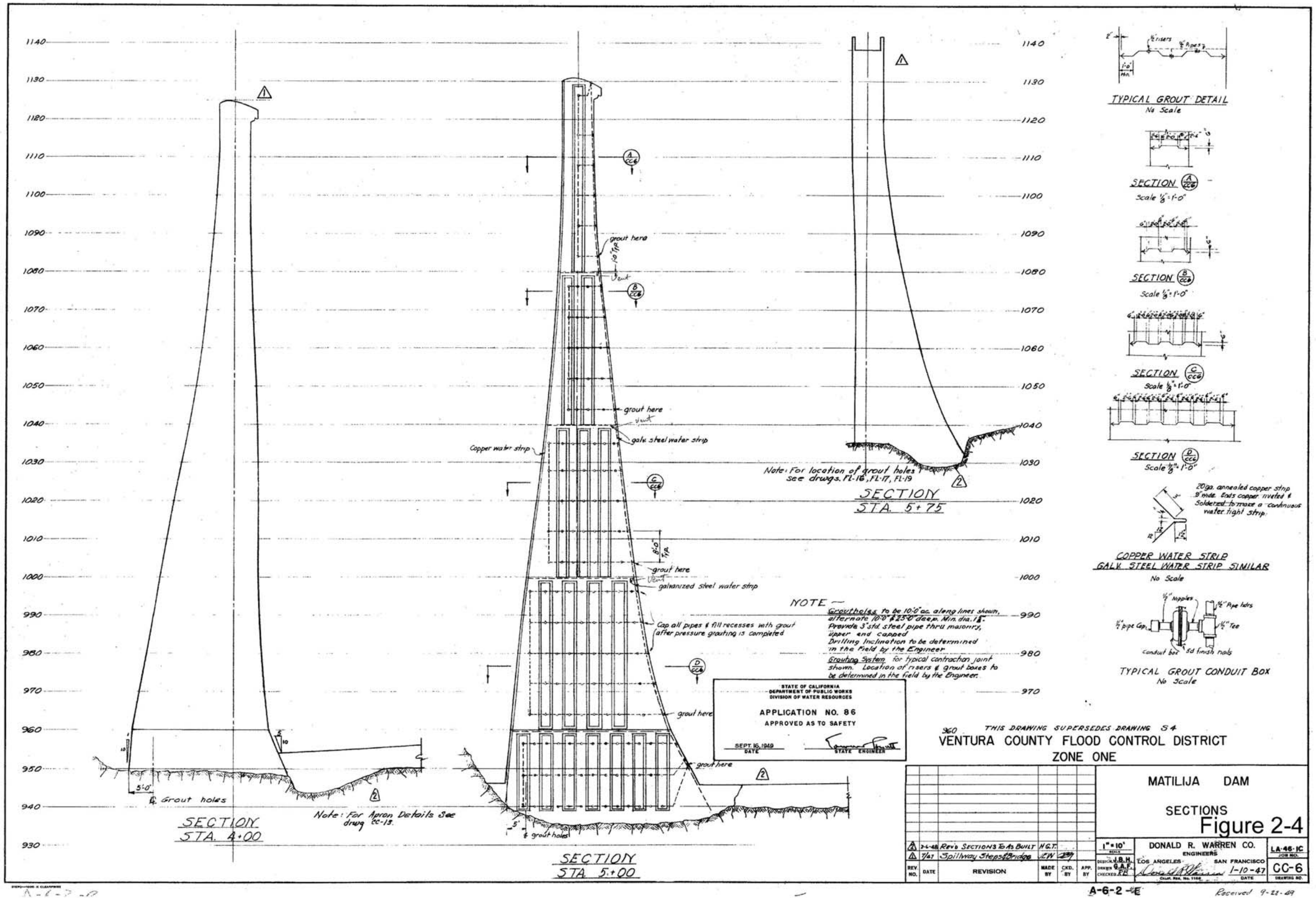

MATILIJA LAKE
N
TARGET K
TARGET L
TARGET J
TARGET I
FACE OF DAM
TARGET F
TARGET G
TARGET H
TARGET E
TARGET D
TARGET C
TARGET B
TARGET A
PILLAR 1
PILLAR 2
PILLAR 3
PILLAR 4
PILLAR 5
PILLAR 6
PILLAR 7
LEFT ABUTMENT
RIGHT ABUTMENT
UPSTREAM
DOWNSTREAM
BASELINE
MATILIJA CREEK
PLAN
LOCATIONS OF TRIANGULATION TARGETS AND PILLARS
1 :800
METRIC
RIGHT ABUTMENT
LEFT ABUTMENT
SPILL ELEVATION= 1095'
D.H. 1R
D.H. 2AR
D.H. 2R
D.H. 3R*
D.H. 1L
D.H. 2AL
D.H. 2L
D.H. 3L*
* NOW ABANDONED
(-) COMPRESSION
(+) TENSION
FACE ELEVATION
LOCATIONS OF TRIANGULATION TARGETS AND STRAIN GAGE DRILL HOLES
NO SCALE
SYMBOL LEGEND
PILLAR
TRIANGULATION TARGET ON FACE OF DAM
STRAIN GAGE DRILL HOLE (D.H.)
ERROR ELLIPSE FROM ORIGINAL AUGUST, 1991 SURVEY (SHEETS 2 AND 3)
LAST POSITION OF TRIANGULATION TARGET (SHEET 2 AND 3)
NOTES REGARDING THE HORIZONTAL DEFORMATION MONITORING
ORIGINAL SURVEY
For this report, the original field survey to monitor horizontal deformation of Matilija Dam was done August, 1991.
SCHEDULE OF RESURVEYS/OBSERVATIONS
Since the original field survey, quarterly resurveys have been scheduled for each March, June, September, and December. For the original field survey and all resurveys, observations were made from each of the seven pillars below the dam to all visible targets on the face of the dam and to the other six pillars. A minimum of five targets are visible from each of the pillars.
BASELINE
The Matilija Dam baseline is defined as a straight line from the original August, 1991 position of Pillar 5 to the original August, 1991 position of Pillar 7.
MOVEMENTS
Right abutment/left abutment movement is defined as the difference in horizontal position of a target or pillar from its orignial August, 1991 position and any later position, measured parallel with the Matilija Dam baseline.
Upstream/downstream movement is defined as the difference in horizontal position of a target or pillar from its original August, 1991 position and any later position, measured perpendicular to the Matilija Dam baseline.
COUNTY OF VENTURA – PUBLIC WORKS AGENCY
VENTURA COUNTY FLOOD CONTROL DISTRICT
MATILIJA DAM
HORIZONTAL DEFORMATION – AUGUST 1991 THRU DECEMBER 2001
PROJ. NO. 11011
SHEET 1 OF 5
Date Revised: 01/25/2002

Figure 2-3


SECTION STA. 4+00
SECTION STA. 5+00
SECTION STA. 5+75
TYPICAL GROUT DETAIL
No Scale
Note: For Apron Details See drwg CC-13.
Note: For location of grout holes see drwgs. FL-16, FL-17, FL-19
Grout holes
grout here
Copper water strip
galv. steel water strip
galvanized steel water strip
Cap all pipes & fill recesses with grout after pressure grouting is completed
NOTE —
Groutholes to be 10'-0" o.c. along lines shown, alternate 10'-0" & 25'-0" deep. Min. dia. 1⅝".
Provide 3" std. steel pipe thru masonry, upper end capped
Drilling Inclination to be determined in the Field by the Engineer
Grouting System for typical contraction joint shown. Location of risers & grout boxes to be determined in the field by the Engineer.
COPPER WATER STRIP
GALV. STEEL WATER STRIP SIMILAR
No Scale
TYPICAL GROUT CONDUIT BOX
No Scale
STATE OF CALIFORNIA
DEPARTMENT OF PUBLIC WORKS
DIVISION OF WATER RESOURCES
APPLICATION NO. 86
APPROVED AS TO SAFETY
SEPT. 16, 1949
DATE
STATE ENGINEER
THIS DRAWING SUPERSEDES DRAWING S4
VENTURA COUNTY FLOOD CONTROL DISTRICT
ZONE ONE
MATILIJA DAM
SECTIONS
Rev'd Sections To As Built
Spillway Steps & Bridge
REVISION
DONALD R. WARREN CO.
ENGINEERS
LOS ANGELES
SAN FRANCISCO
1-10-47
LA-46-IC
CC-6
A-6-2-E
Received 9-22-49
Figure 2-4